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Cheero Point landslide

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ABSTRACT: In 1989 the Pacific Highway at Cheero Point, near Mooney Mooney, was closed by the Roads and Traffic Authority (RTA) of New South Wales because it was considered unsafe. A site investigation was performed which revealed movement of the embankment supporting the Highway at this location. However, there was no observed movement of the Highway itself. The RTA recently requested another investigation of this project with the view of re-opening the road. A number of possible remedial schemes were costed, a geotechnical mapping survey performed and an environmental study of the embankment carried out. Also, a poster presentation was produced of the embankment for public viewing explaining the conclusions of the analyses. The final remedial scheme is currently being designed; a piled bridge founded on rock.

1 INTRODUCTION

During the 1960's a section of the Pacific Highway near Cheero Point, NSW, was relocated as part of the Sydney to Newcastle Freeway construction. A rockfill embankment was constructed on the existing slope as part of the relocation works. The fill was placed over colluvium and residual soil overlying sandstone.

Instability of the slope was first reported in 1974, and movement increased significantly following heavy rainfall in early 1975. The embankment failed through fill and underlying colluvium leaving an extensive backscarp. However, lateral movement was limited. No remedial measures were undertaken at this stage.

Since 1975 the embankment has gradually moved and restabilized several times. This creep style movement has been driven by prolonged heavy rainfall. Geotechnical surveys and analyses were conducted to determine the extent of the slope instability and the causes and mechanisms of failure. In 1989 the Highway was considered unsafe by the RTA and was closed. Various remedial schemes involving stabilization of the Highway and slope were considered.

By 1992, with the onset of drier conditions, movement again ceased. A Value Management Meeting was held in April 1993 to reassess the situation and involve all interested parties. As a result of this meeting the extent of potential movement of failure material was assessed. The magnitude of this movement affected both the nature and cost of the remedial measures.

2 SITE DESCRIPTION

The site covers a length of approximately 500m on a north-east facing slope adjacent to the Highway. It is located near Cheero Point, NSW, and is approximately 53km north of Sydney and 3km north of the Hawkesbury River Bridge.

The slope extends down from the Highway to a local thoroughfare, Cheero Point Road, which is adjacent to Mooney Mooney Creek. There are existing dwellings in the area and several lots which have subsequently been cleared. At the base of the embankment is a children's playground, now closed by the RTA for safety reasons.

Between the Highway and Cheero Point Road the slope varies in length from approximately 30 to 60m. The Sydney to Newcastle Freeway is situated upslope and almost parallel to the Highway at this location. Figure 1 shows a plan of the site.

3 SITE INVESTIGATION

Sub-surface and groundwater conditions were assessed and ground movements monitored to determine possible failure mechanisms.

3.1 Sub-surface conditions

It can be seen from Figure 2 that a typical geological sequence comprised end-dumped boulders overlying granular fill along the upper part of the slope. This granular fill was found to consist generally of loose to medium dense silty sand and gravel with occasional boulders.

Below this fill was colluvium and residual soil. This comprised a firm to stiff silty clay layer. The boundary between colluvium and completely weathered rock (residual soil) was almost impossible to distinguish. Under this soil layer was medium-grained Hawkesbury Sandstone. This bedrock was found to be thickly bedded to massive, and exhibited distinct horizontal, as well as cross bedding, dipping up to 20°. Near vertical joints existed in the sandstone rock.

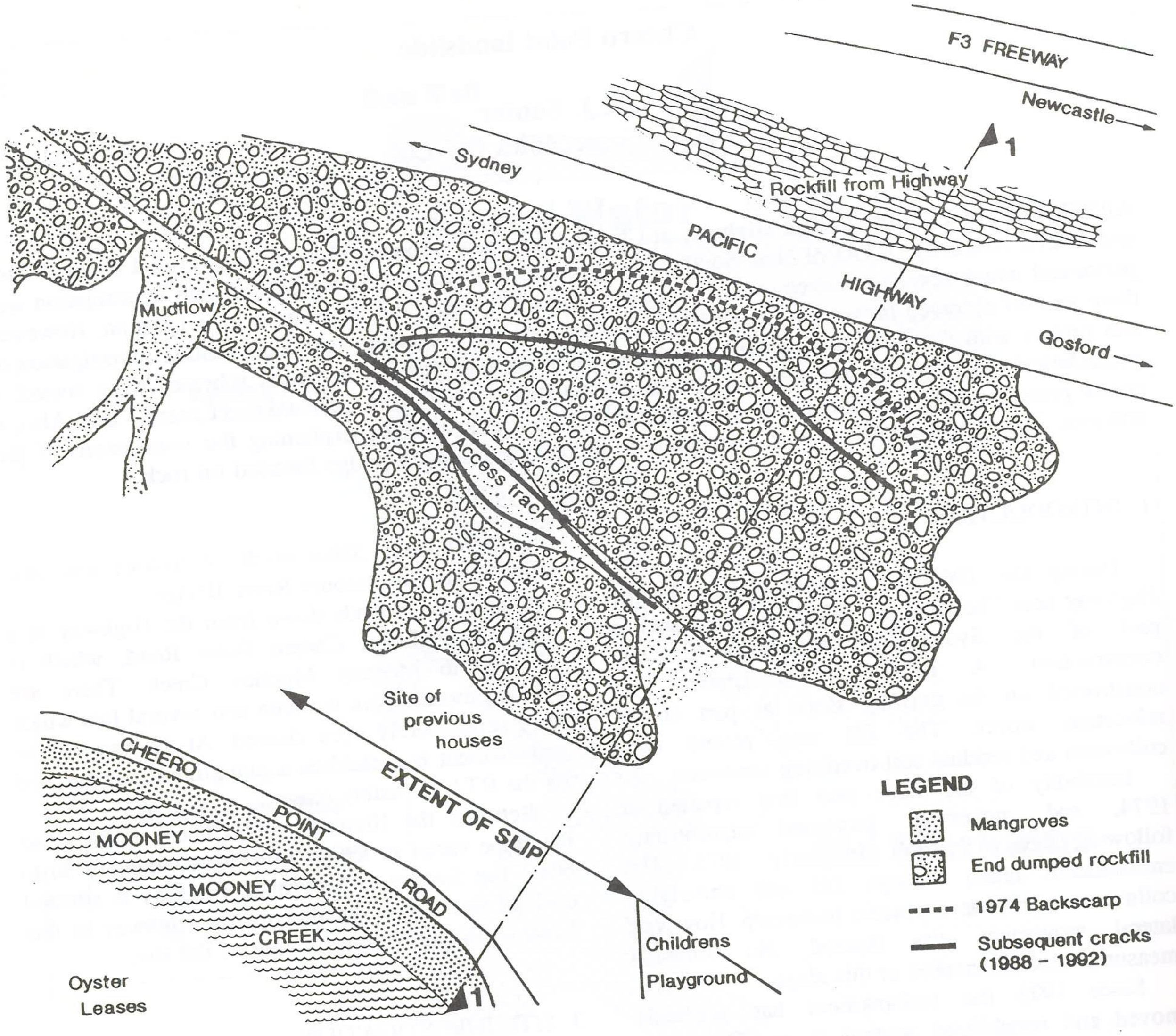


Figure 1 Site plan

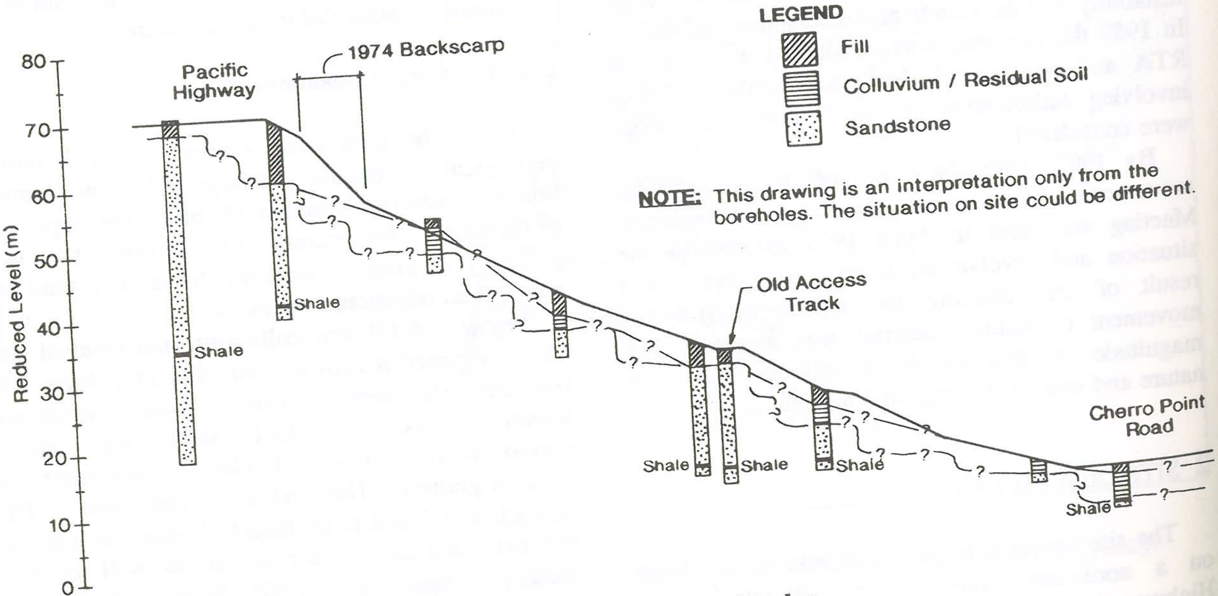


Figure 2 Section 1, taken from site plan (showing typical geological sequence)

3.2 Groundwater conditions

Readings from the standpipes installed across the slope indicated that generally the water table was situated near the base of the fill layer and showed little variation after rainfall. However, water levels were not monitored during heavy rainfall. Peak levels were recorded using bucket piezometers.

3.3 Ground movement monitoring

Surface movement pegs were placed as part of the initial survey and their positions have since been regularly checked. Geotechnical mapping of the site was performed. Repeat surveys indicated changes in the site and ongoing movement of soil masses.

3.4 Possible failure mechanisms

From monitoring and analysis performed in 1989 it was apparent that ground movements took the form of shallow slides within the fill and colluvium overlying rock. Movements were continuing at a rate which was probably controlled by rainfall. Figure 3 shows a typical slip mechanism. The back scarp and toe bulge illustrated were evident on site.

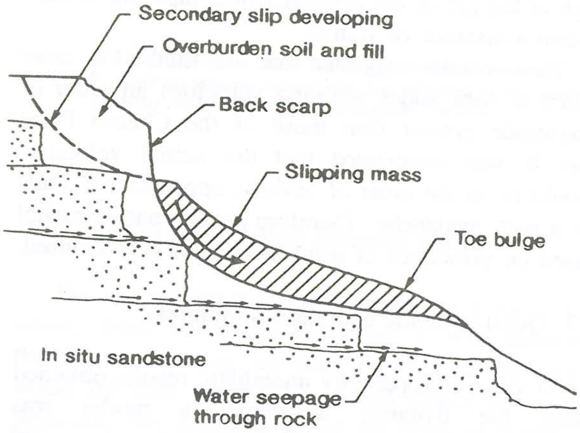


Figure 3 Typical slip mechanism

4 VALUE MANAGEMENT WORKSHOP

This workshop was held in April 1993 at Gosford and was attended by the Lord Mayor and representatives from the RTA, Arup Geotechnics, the police force, the ambulance service, and the local community. The results of the Workshop were presented in a report prepared by Bushell (1993).

The outcome was that the Highway should be reopened. The selected remediation schemes were, for the Highway and slope respectively, a bridge with piers on rock and catch fences erected across the slope to retard dislodged boulders.

Signposts and New Jersey kerbs have been installed along Cheero Point Road to warn residents of the danger. A poster presentation was produced for Gosford Council and the local community explaining, in non-geotechnical terms, the various issues concerning the slope.

5 SLIP DYNAMICS

The Value Management Workshop determined that the Highway should be reconstructed on a bridge to avoid any possibility of slip failure. Any design of remedial works on the slope is dependent on the extent and possible consequences of landslip movement. If movement is excessive the following problems arise:

- risk of pedestrians and traffic being buried, and Cheero Point Road being closed;
- playground at the embankment base could be engulfed and therefore remains closed;
- damage to oyster leases and mangroves in Mooney Mooney Creek.

A slip dynamic analysis was performed to determine the ultimate position of the slipped mass of material following a landslide. An extensive literature survey was undertaken to assess appropriate analysis techniques. The study revealed a general lack of published literature in this field. Two analysis techniques were used: first, a true dynamic method obtained from the literature; and second, a quasi-dynamic equilibrium model derived from first principles. Quasi-dynamic behaviour refers to freezing the analysis in incremental time steps and applying a static equilibrium model. Both methods provided indicative, rather than accurate predictions of slip behaviour.

5.1 Dynamic method

This was based on theory given by Van Gassen and Cruden (1989). They describe how to determine the distance a slip moves past the original toe position, for a known slip velocity.

There were three possible sliding mechanisms:

- slide with constant mass
- slide with linearly changing mass
- slide with exponentially changing mass

The *runout length*, L, is defined as the distance from the slope toe to the centre of gravity of debris, after a landslide. L can be determined for each of the above sliding mechanisms. From Van Gassen and Cruden (1989), these are given as:

constant mass

$$L = -\frac{V^2}{2g(\sin\theta - \tan\phi\cos\theta)}$$

linear deposition of mass

$$L = -\frac{3}{2}\frac{V^2}{2g(\sin\theta - \tan\phi\cos\theta)}$$

exponential deposition of mass

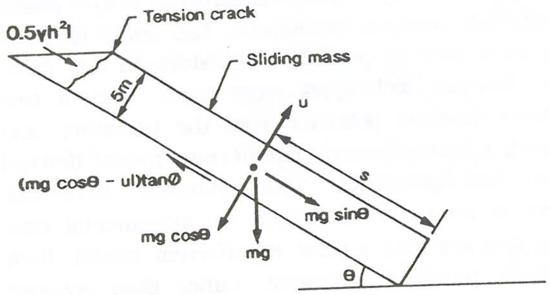
$$L = -1.93\frac{V^2}{2g(\sin\theta - \tan\phi\cos\theta)}$$

where V = velocity of sliding mass.

To use this method the velocity was required, although its determination is not covered in the paper by Van Gassen and Cruden (1989). A simplistic approach using Newton's Laws of Motion was adopted. Figure 4 shows the assumed forces acting on the sliding mass. These were used in the velocity calculations, assuming limiting equilibrium:

$$(mg\cos\theta - ul)\tan\phi = mg\sin\theta$$

ϕ was taken to be constant at 30° because the material was assumed to be at residual strength, having suffered significant movement already.



ASSUME:

(mg cosθ - ul)tanØ = mg sinθ

m = mass of sliding material
g = acceleration due to gravity
θ = slope angle
tanØ = friction coefficient
l = length of slope parrallel to highway
u = pore water pressure
h = depth of tension crack
s = depth of slip material
γ = specific weight of water

Figure 4 Force equilibrium

The driving force, F, causing movement of the mass was assumed to arise from hydrostatic pressure within a tension crack along the upper edge of the slope. A constant supply of water was assumed to maintain the driving force. Since this supply was unlikely to be available in practice, this approach was believed to be conservative. See Figure 4.

$$F = \frac{1}{2}(\gamma h)hl$$

γ = specific weight of water

h = height of moving mass (assuming a full tension crack).

Acceleration, a, of the moving mass was then found from Newton's 2nd Law of Motion, F = m a.

From simple body dynamics:

$$V^2 = U^2 + 2as$$

where U = initial velocity (= zero)

s = distance moved

Therefore the final velocity, V, of the moving mass was determined. From the calculations it was found that the velocities were low. With these velocities, calculated runout lengths varied widely. In the constant mass model the lengths appeared high, around 50m. The corresponding value (≈ 5m) in the linear deposition model appeared too low. In the exponential deposition model the runout length, at approximately 10m, was more realistic. However, the estimated shape of the slip debris was less satisfactory. The model suggested the exponential deposition would result in slide debris some 50m high at the toe of the original slope, tapering to zero within a distance of 10m.

These results suggested that this method is more suited to rock slides attaining velocities an order of magnitude greater than those of the Cheero Point slip. It was anticipated that the actual velocities should be in the order of 5m/s as opposed to 70 m/s for a rock avalanche. Therefore an alternative model based on principles of solid mechanics was devised.

5.2 Quasi-dynamic equilibrium method

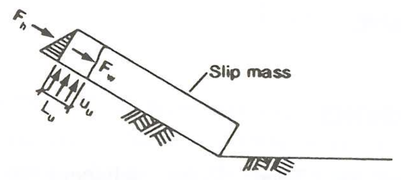
In view of apparently unrealistic results obtained using the dynamic technique, a model was developed where the equilibrium of the slope was assessed over incremental time intervals. Forces acting on the sliding mass were determined for these time increments and hence accelerations and velocities of the sliding block were determined. These values were then used to calculate forces and movements for the next time increment.

Figure 5 shows the forces assumed to be acting on the sliding mass. It was assumed that the slope was in limiting equilibrium and that movement occurred as an intact block of material.

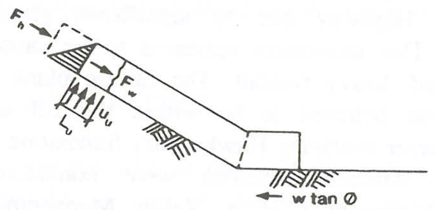
The driving force was assumed to be provided by two components. First, a driving hydrostatic force, F_h, at the top of the slope:

$$F_h = \frac{1}{2}\gamma h^2$$

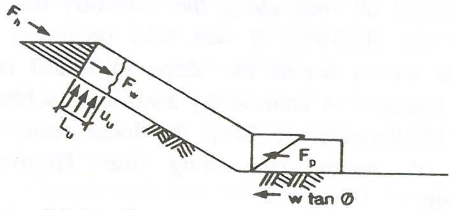
It was assumed that there was sufficient water to maintain this head indefinitely. As for the dynamic model, this assumption was considered conservative.



1. Initial state.



2. Slip mass moves down slope. Friction along base slows down slip. Slip mass halts when w tan Ø exceeds driving force.



3. Friction Force at base exceeds passive resistance, F_p. Failure wedge rides over top of base block until passive resistance again exceeds friction.

Figure 5 Quasi-dynamic model

The second assumed component was a force, F_w, arising from porewater pressures. It was assumed that cracks opening up near the top of the slope would fill with water. This would induce porewater pressures over a length, L_u, of the slope as shown in Figure 5. The maximum head of water would equal the thickness of the sliding material. The weight of this block of material over the length, L_u, would provide a component of the driving force, F_w

$$F_w = mg\sin\theta - (mg\cos\theta - u_u)\tan\phi$$

where $m = h\,L_u\,\rho$
h = block height
L_u = height of pore water pressure

Therefore the total driving force, $F = F_h + F_w$

The total resisting force, F_{res}, was then calculated by determining the acceleration of the sliding mass and hence the velocity and displacement. This displacement of the sliding mass yielded the slip length and ultimately the resisting force. This force then yielded a new value of deceleration due to base friction. This deceleration was then used to determine a new acceleration and the above process was repeated. This iterative process was continued until the velocity of the sliding mass became zero. The theory is outlined below.

At time t = 0:

$a_0 = 0$; $V_0 = 0$; $S_0 = 0$; $D_0 = 0$

At time, t:

total length of block	$= L_o$ (constant)
total driving force, F	$= F_h + F_w$ (constant)
mass of block	$= m$
acceleration	$= a_t = (F/m) - a_{res}$

a_{res} = deceleration due to frictional resistance at base

At time t-1:

displacement of sliding mass	$= s_{t-1}$
length of block remaining on slope	$= L_{t-1} - s_{t-1}$

To determine the deceleration, a_{res}, due to base friction, the resisting force, F_{res}, was first calculated.

The displacement, s_t, and the velocity, V_t, at time t, were calculated from the Laws of Dynamics:

$$s_t = V_{t-1}\Delta t + \frac{a_t \Delta t^2}{2}$$

$$V_t = V_{t-1} + a_t \Delta t$$

where Δt = time interval

The slip distance, D, is defined as the distance from the toe of the slope to the toe of the slip. D was calculated at any time, t, from:

$$D_t = D_{t-1} + s_t$$

From these calculations the resisting force, F_{res}, was determined:

$$F_{res} = W_t \cos\beta \tan\phi - W_t \sin\beta$$

where β = slope of base
ϕ = angle of friction along base
$W_t = D_t\,h\,\gamma$

$$a_{res} = \frac{F_{res}}{m}$$

where $m = L_o\,h\,\rho$

The slip distance varied from 5 to 25m and was dependent on the porewater pressures developed along the failure plane. Control of the surface water above and on the slope was therefore considered an important measure in limiting the runout of a slide.

In the analysis, a point was reached during the slide when frictional resistance along the base exceeded passive resistance at the slope toe. At this stage passive failure occurred if forces were large enough. Material moving down the slope rode over stationary material until a sufficient height was achieved for the passive to once again equal or exceed the total resistance along the base. This phenomenon did not affect the total resistance to sliding as this value was dependent on the mass of material at the base of the slope and not its shape.

5.3 Preferred method

From previous observations, movement of the Highway embankment was expected to be gradual or to occur in short slip movements after prolonged, heavy rainfall. Large scale mud slides or rapid failure of the slope was not anticipated. Results from the quasi-dynamic equilibrium model closely reflected actual movements that have already taken place on the site.

The true dynamic model proved inappropriate for this site because it:

- assumed the entire slope failed and was deposited at the base of the embankment;
- was developed for steep slopes with high velocities (in the order of 70m/s);
- assumed the sliding mass achieved a terminal velocity before reaching the base of the slope;
- determined runout lengths which were very sensitive to small changes in velocity.

5.4 Effects of potential landslide

For potential landslides it was anticipated that the velocity of the moving material would be low, in the order of 5m/s. Movement was expected to be gradual with slip distances of approximately 5 to 25m, depending on the porewater pressures developed along the failure plane.

A landslide would have several impacts on the local environment, depending on the slip severity. If the landslide occurred instantaneously, for example as a single movement, and the slip distance was 25m, material would be deposited in Mooney Mooney Creek. This would damage and possibly kill the mangroves. However, it was believed that the deposited sediment would have an insignificant impact on the oyster leases. However, Cheero Point Road, the only access for local residents, would become blocked. The landslide material might also create a pedestrian and vehicular hazard. The children's playground may also be affected along one boundary. This instantaneous landslide model was believed unrealistic based on site observations and recorded ground movements.

From observations and site monitoring, it was anticipated that movement of the slope would be gradual rather than instantaneous. The above-mentioned environmental impacts were therefore unlikely to occur and remediation measures would be less extensive.

6 CONCLUSIONS

The slope at Cheero Point, adjacent to the Highway, was gradually slipping. In 1989 the RTA closed the Highway due to significant ground movement. This movement appeared to be caused by prolonged, heavy rainfall. The failure plane of this slip was believed to be within the fill and colluvium layer overlying Hawkesbury Sandstone.

Various remedial schemes were considered, costed and presented at a Value Management Workshop. The outcome of this Workshop was a decision to re-open the Highway and construct a bridge founded on rock along the currently unsafe section of the Highway. It was also proposed to erect catch fences across the slope to retard any dislodged boulders. A community awareness scheme was also implemented to keep the local residents informed of issues concerning the Highway embankment.

7 REFERENCES

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